# 1 Introduction

## 1.1 Purpose

The proposed trench depression passes through the downtown City of Reno core and passes adjacent to or under some structures. During the construction of this trench system, eliminating excessive settlement of these adjacent structures is a paramount consideration of overall trench performance. To minimize these settlements, underpinning techniques must be examined.

Upon examination, the feasible methods are presented with historical structural performance, a brief description of typical construction details, and associated approximate construction costs. For the purposes of this report, Mass Concrete, Minipiling, and Soil Grouting techniques are examined. The following document contains a discussion of each of the underpinning techniques previously proposed.

## 1.2 Setting

The entry building to the Fitzgerald's casino supports the north end of the Rainbow Pedestrian Bridge and is used as an entrance for the Fitzgerald's casino. The building is a two-story steel frame with a mezzanine area in the western half of the building. The building is located directly adjacent to the proposed trench wall. The floor system is composed of a 3½" lightweight concrete slab over a 2" x 20 gauge metal deck. The building measures 67'-4" (East-West) by 20' (North-South). The existing Southern Pacific Railroad alignment passes under the Rainbow Pedestrian Bridge directly adjacent to the entry building.

The foundation system for the entry building, in the location of the trench, is a grade-beam at a depth of approximately 4-feet below original ground. The grade beams support the steel wide-flange columns. While the length of the building is 67'-4", the length of the footing directly adjacent to the railroad tracks that would requiring underpinning is 72'-4".

The depth of the proposed wall trench at the Rainbow Pedestrian Bridge is approximately 32-feet (measured from the original grade to the top of the reinforced concrete invert slab). The approximate thickness of the reinforced concrete slab is 5-feet. The underpinning solution is anticipated to extend below the reinforced concrete slab to be embedded approximately 4-feet into the jet grout layer. Thus, overall underpinning excavations adjacent to the entry building will be approximately 38-feet below the base of the existing foundation for a total depth of 41-feet, measured from the original ground. Adding to the complexity of the underpinning solution, the depth to groundwater is approximately 27-feet. Therefore, of the total excavation depth of 41-feet, approximately 14-feet will be conducted below the groundwater table.

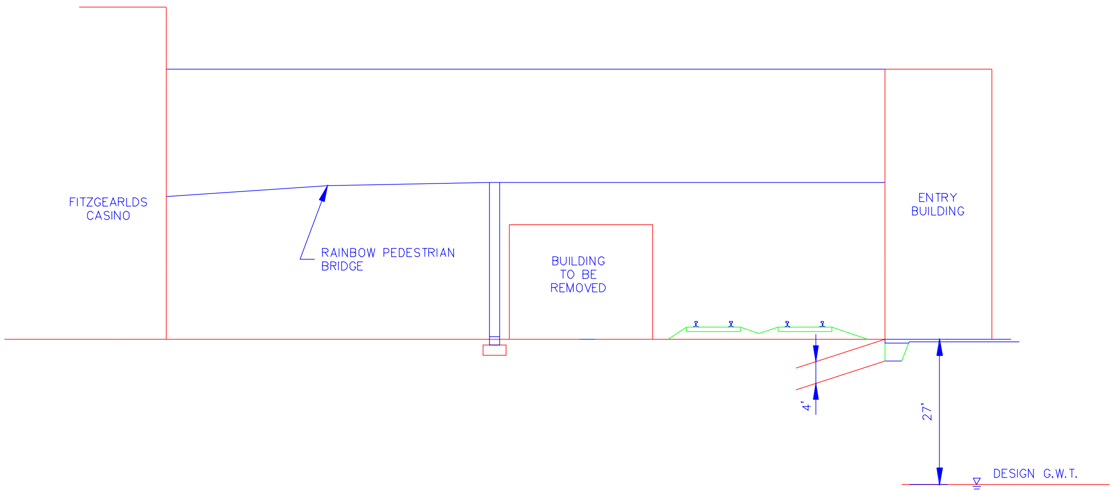


**Figure 1 Existing Conditions**

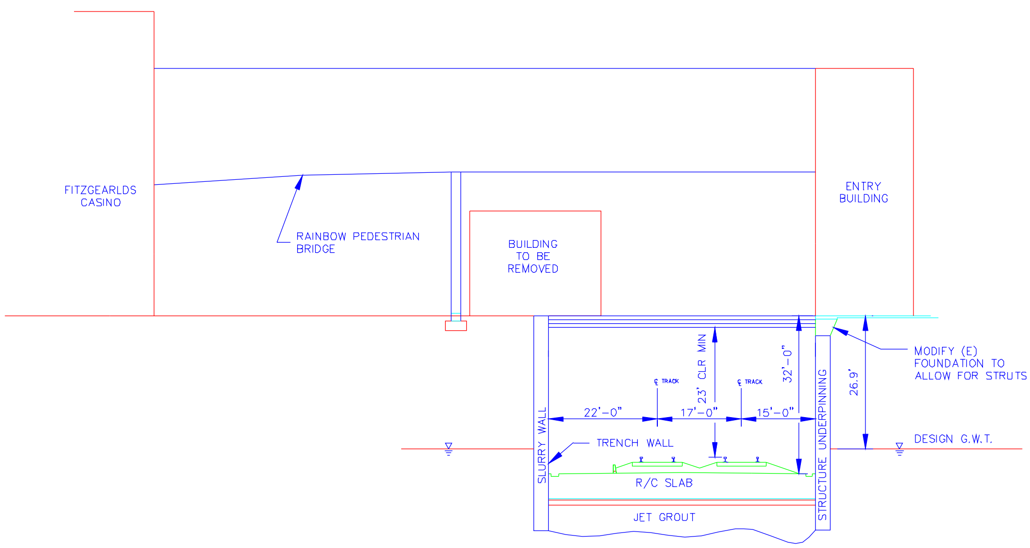


**Figure 2 Proposed Section**

# 2 General Concepts

Each underpinning technique was challenged against one another in the areas of 1) soil applicability, 2) design and construction feasibility, and 3) cost of construction. Each of these categories was explored in the detailed sections of this report.

To determine the best practical solution for the underpinning of the entry building for the Rainbow Pedestrian Bridge, an analysis of soil applicability was examined. The result of this analysis was used in an initial screening of each underpinning method. Since the soil, in the City of Reno vicinity, is comprised of loose to dense sands, and gravels, only methods that are compatible with these conditions were examined.

The design and construction feasibility of each method was evaluated by performing conceptual calculations and/or collaboration with foundation underpinning experts. This information was necessary to develop enough information to gain an understanding of a possible solution in final design. Only enough information was determined to address feasibility and construction costs associated with each method. Specifically, the design criteria that was used to determine feasibility included an examination into whether the final conditions would provide 1) elimination of groundwater seepage, 2) adequate lateral support for soil and groundwater forces, and 3) suitable solutions to support the structure in both the temporary and final conditions. Furthermore, construction related concerns were addressed by ensuring the recommended methods of construction 1) had a proven history of success in similar applications, 2) provided for safe and efficient progress, and 3) were possible in the City of Reno.

The final construction costs were estimated using the results from the conceptual calculations and collaboration with experts. An estimated total cost of the construction was determined for the entire underpinning effort for the entry building to the Rainbow Pedestrian Bridge, except the foundation modifications required to install the proposed trench struts. The preliminary costs presented in this report reflect the costs associated to a finished trench wall that resists vertical loads and provides a positive groundwater barrier. In the final analysis, conducted to determine the preferred method of underpinning, these costs were used to rank the proposed methods. Specifically, any option with an estimated construction cost in excess of $1,000,000 was eliminated from contention. This number was used as a basis as it is approximately half the assumed replacement value of the structure.

All proposed underpinning techniques were analyzed using the specific screening criteria presented above. Each of these analyses is presented in the detailed sections of this document. The conclusion of this document summarizes each criterion described above and provides a recommendation for underpinning construction.

# 3 Mass Concrete

## *3.1 Methodology*

The concept of mass concrete has been used for decades. Mass concrete underpinning is accomplished by excavating a segmental trench under the existing foundation to various depths (up to 60-feet). After excavation, the new hole is filled with unreinforced or reinforced concrete and another hole is excavated a distance away from the first (see diagram below). This pattern continues, exposing 20% or less of the existing foundation at a time, until the entire structure is completed. This technique is typically employed on shallow foundations to lower the effective base of the structure, reducing differential settlements. However, with proper reinforcement details and construction practices, it may be used to extend foundations to a much greater depth.

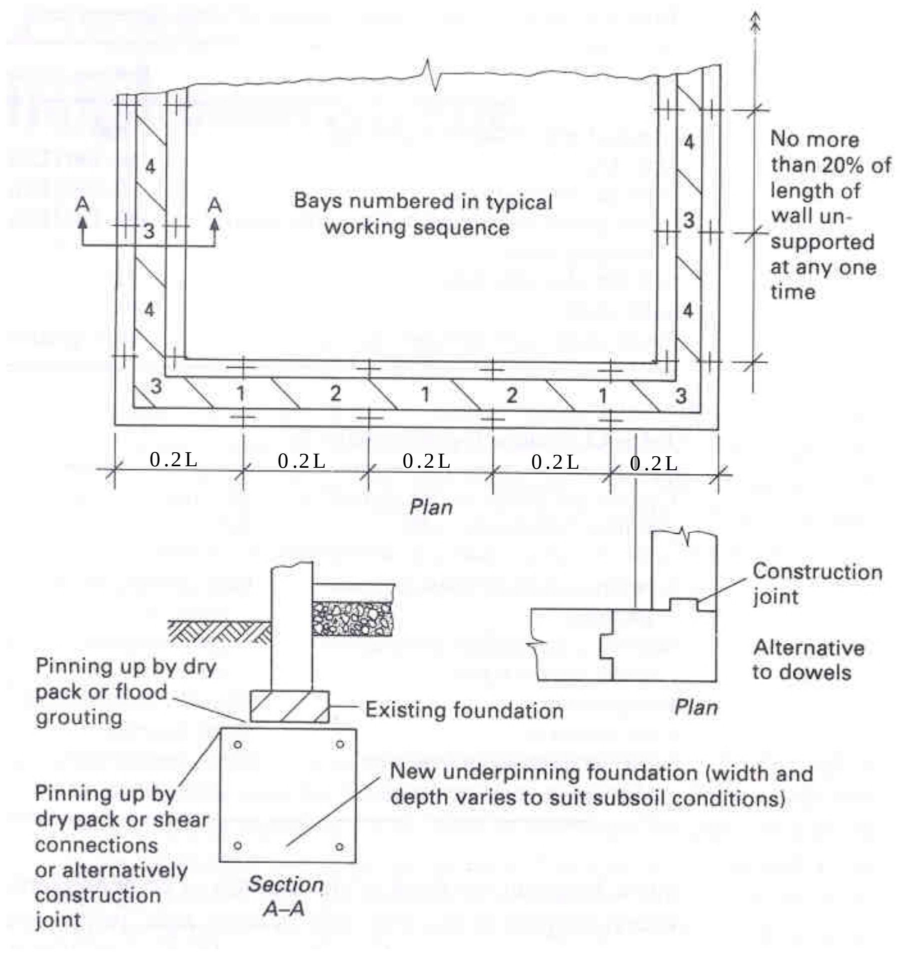


**Figure 3 Mass Concrete Underpinning**[1]

[1] Bullivant and Bradbury, *Underpinning*

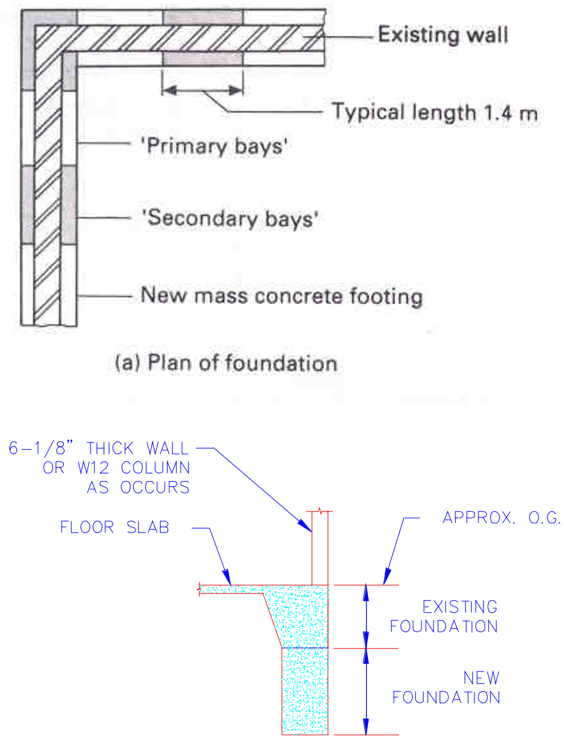


**Figure 4 Mass Concrete**[2]

### *3.2 Applicability to Soil Conditions*

Mass concrete is applicable in any soil condition, provided that the bottom of the excavated pit is founded on competent material. The materials within the City of Reno (*Geotechnical Engineering Report, Proposed Reno Railroad Corridor EIS*, Reno, NV, Prepared by Kleinfelder, May 19, 2000) may be classified as competent soil from 2- to 3-feet below the surface to great depths. However, the presence of a relative high groundwater table would increase the difficulty of this construction. Additionally, the lack of cohesion in the soil may necessitate the use of slurry during the excavation procedure to maintain an open excavation.

### *3.3 Design and Construction Feasibility*

The construction method of Mass Concrete is most similar to the proposed diaphragm walls for the trench system. Therefore, it is anticipated that a 3- to 5-foot wide mildly reinforced concrete section would be able to resist the dead and live loads imposed by the entry building. Furthermore, it is anticipated that the wall of concrete that is produced with this method would create a barrier to water infiltration. Based on these assumptions, this method of underpinning can be designed.

Following conversations with underpinning experts[3], construction of this option is feasible. Similar work has been successfully accomplished throughout the world.

[2] Bullivant and Bradbury, *Underpinning*

[3] Ronald Chapman, V.P., Schnabel Foundation Company, Walnut Creek CA

### *3.4 Cost*

Using past experiences with traditional underpinning and adjusting costs to account for the presence of groundwater and the need for slurry, Mass Concrete construction can be completed for approximately $200 per square foot of trench wall surface[4]. Based on these assumptions, and an approximate area of wall of 3,000 ft$^2$, the total cost of this alternative is $600,000.

### *3.5 Conclusions*

Mass Concrete is typically used for shallow foundations. However, construction detailing and techniques may be modified to make this a feasible option for underpinning the Rainbow Pedestrian Bridge building.

[4] K. Ronald Chapman, P.E., Vice President, Schnabel Foundation Company, Walnut Creek, CA

# 4 Minipiling

## 4.1 Methodology

Minipiling (also know as, Micropiling or Pinpiling) is a technique whereby a shallow foundation is converted to a deep foundation through the installation of small diameter steel piles and anchored to the existing foundation. These piles are typically 5- to 12- inches in diameter and drilled into the soil with small and agile drilling equipment. These techniques are especially useful for larger, heavier structures founded on inadequate soil. The capacity of these piles can be as great as 300,000 pounds per pile in compression. The main contributor to the pile's capacity is skin friction along the surface of the pile.

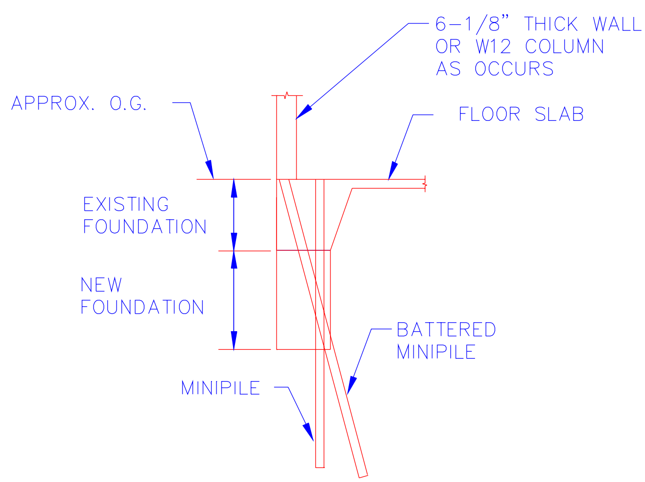


**Figure 5 Minipiling**[5]

## 4.2 Applicability to Soil Conditions

Drilled-in-place elements are suitable in most subsurface conditions. However, in cobbly soil with large boulders or rock, the placement of these elements may prove troublesome. Explicitly, it is anticipated that acceptable production rates will only be obtained by using eccentric down-hole hammer drilling equipment[6]. Since, Minipiling fails to provide a positive groundwater and soil barrier, it serves as a temporary support system to reduce settlements in the structure during excavation. However, additional work will be required to meet the structural performance criteria. These additional construction items must include a wall system to resist lateral forces and provide a positive groundwater barrier.

## 4.3 Design and Construction Feasibility

Design of the Minipiling system poses a challenge that may be surmountable, yet expensive and difficult to construct. The largest engineering challenge for Minipiling is to provide enough lateral support and length of pile to support the structure during the excavation process. During this process, these piles will be

[5] Bullivant and Bradbury *Underpinning*

[6] Rob Jameson, Nicholson Construction Company, Oakland, CA

exposed for a length in excess of 39-feet. Since these piles resist vertical loads through a skin friction mechanism, it will be necessary to install these piles a distance of approximately 40-feet below the lowest point of excavation. Using these values, the piles must be in excess of 79-feet long. Additionally, the unsupported length makes these piles vulnerable to buckling. It may be possible to overcome the buckling vulnerability by installing whalers (horizontal braces) that are anchored into the retained soil with grouted ground anchors. Although surmountable, these issues add considerably to the costs and risk of the completed solution.

Construction of the Minipiling system is initially difficult with the cobbly soils in the City of Reno. Furthermore, excavation, concrete, and reinforcement placement around these piles is a delicate and time consuming operation. These construction and design difficulties render this method undesirable.

### *4.4 Cost*

The cost of underpinning the Rainbow Pedestrian Bridge utilizing Minipiles includes the cost of constructing the piles and a structural diaphragm wall under the existing foundation. This supplemental wall is designed to retain the soil under the building and resist the lateral forces imposed on the wall. The cost of the piles is approximately $200 per foot of pile. Using an approximate length and number of piles, 80-feet and 15-piles, respectively, the cost of the piling system would be $240,000. Additional costs are required to construct the structural wall. The cost of this wall is approximately $80/ft$^2$. Based on 3,000 ft$^2$ of wall surface, the wall cost is $240,000. Adding each of these costs together, the total cost of construction for this option is $480,000, without the use of whalers.

### *4.5 Conclusions*

Based on the requirement to construct a gravity system to address the concerns of building settlement during construction, an additional system to withstand the lateral forces (soil and water), and to provide a positive groundwater barrier, this option is not recommended, because of higher risk and high costs.

# 5 Soil Grouting

## *5.1 Methodology*

Soil Grouting procedures are employed to improve the internal shear and compressive strength of in-situ soils. Soil Grouting is typically installed by drilling small diameter holes (4- to 8-inches) below the existing foundation and injecting chemical or cement grouts into the soil. The columns of grouted soil are used to transfer the compressive forces of the structure to a deeper location. In the case of spread (or strip) footings, the grout columns must be installed contiguous to each other under the entire footing. After the grout is injected, trench excavation can occur adjacent to the grouted columns without disturbing the existing foundation.

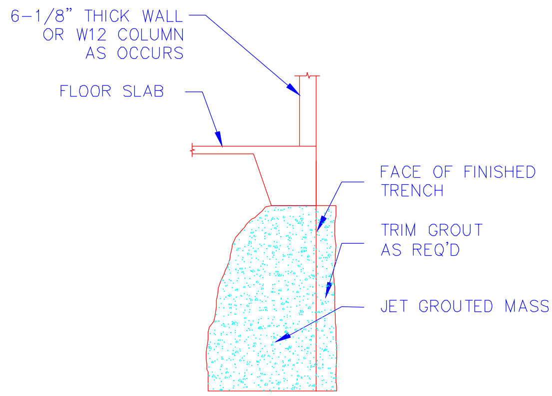


**Figure 6: Jet Grout with Diaphragm Wall**

## *5.2 Applicability to Soil Conditions*

Soil Grouting is accomplished through various installation methods, including permeation, injection and jet placement. The City of Reno soils are applicable to all grout installation methods. The most favorable method for Soil Grouting is jet grouting.

Jet grouting can be used for a wide range of soil types, but special care is required when dealing with very stiff and/or highly plastic clays. Local obstructions, such as boulders, can often be bypassed or encapsulated into the jet-grouted soil mass.

Variations in soil fines content, gradation, and density are likely to result in irregularities in the radius of the jet-grouted columns, thereby increasing subsequent excavation difficulties. Furthermore, modifications to typical jet grout underpinning with grouted columns will be required. These modifications include constructing contiguous series of grouted columns along the entire length of the existing footing.

The final excavated trench will need to be a positive groundwater barrier. However, traditional grout underpinning concentrates on developing cemented soil columns for support. With the application required at this location, the columns will have to be installed continuously to form a continuous mass of soil down the length of the structure.

### *5.3 Design and Construction Feasibility*

The design of grouted underpinning poses three distinct engineering challenges. The first challenge is the ability to provide a positive groundwater barrier. The second difficulty is protecting the finished facing from the climactic elements. The third problematic construction procedure is to finish the wall surface despite the soil conditions.

Although creation of a watertight solution with grouting is possible, it is challenging in loose soils. A proposed solution is to fortify the grouting just below the groundwater table. This fortification is accomplished by providing a wide grouted soil mass at the base of the underpinning (Figure 6). Additionally, the exposed face of the grouted soil requires an engineered solution.

The preferred method to grout under this structure is through the use of jet grouting[7]. Jet grouted soils weather poorly and require an additional facing material to protect it from degrading. Facing material is typically shotcrete. However, in the City of Reno, concerns of frost heave between the two layers (jet grout and shotcrete) negate the use of any facing. Therefore, from a design perspective this underpinning solution is not recommended.

Although the construction of the wall in soils containing large boulders and cobbles is manageable, providing a finished surface to the wall given these soil conditions is difficult. Cobbles and boulders crossing the finished plane of the wall will require the use of a jackhammer or similar equipment for removal. Extremely large boulders might present a water seepage problem upon removal and these voids must be patched.

### *5.4 Cost*

Based on grouting costs published in the *Draft Alternative Wall and Invert Report*, July, 2000, prepared by Nolte Associates, Inc., treated material is approximately $250 per cubic yard[8] (adjusted for labor intensity). Using an average thickness of 8-feet and a surface area of 3,000 ft$^2$, the total cost of this alternative is approximately $222,000.

### *5.5 Conclusions*

Soil Grouting is the least expensive of all the proposed options in this report. However, due to the unfavorable weathering concerns of the exposed surface and the unpredictability of this method to provide a positive groundwater barrier, Soil Grouting is not recommend.

## 6 Conclusion

The proposed solutions examined in this report were 1) Mass Concrete, 2) Minipiling, and 3) Soil Grouting.

Mass Concrete provides a positive groundwater barrier, resists lateral forces, and is feasible in the City of Reno. The total estimated construction costs associated with Mass Concrete are approximately $600,000.

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[7] Draft Wall and Invert Analysis Report, Prepared by Nolte Associates, Inc., July, 2000

[8] Alan R. Ringen, P.E, Hayward Baker, Santa Paula, CA

Minipiling ($480,000) exposes the project to undesirable risks associated with large unsupported lengths and construction access limitations. Therefore, Minipiling was eliminated from the list of viable alternatives.

Although the least expensive alternative ($222,000) and a plausible solution for groundwater seepage, Soil Grouting was eliminated from contention in this analysis. The undesirable weathering concerns of the exposed face of the grouted soil mass were the contributing factors in the elimination of this option.

Based on engineering and construction criteria, both Minipiling and Soil Grouting were eliminated from the recommendations of this report. Therefore, the remaining option, Mass Concrete, is the recommended underpinning solution of the Rainbow Pedestrian Bridge.

# 7 Bibliography

1. Bullivant, Roger A. Bradbury, H.W. Underpinning A Practical Guide. Blackwell Science Ltd., 1996
2. Thornburn, S., Hutchison, J.F. Underpinning, Surrey University Press, 1985
3. Goldberg, D.T., Jaworski, W.E., Gordon, M.D. *Report No. FHWA-RD-75-128 Lateral Support Systems and Underpinning, Vol. I. Design and Construction.* Federal Highway Administration, 1976

# 8 References

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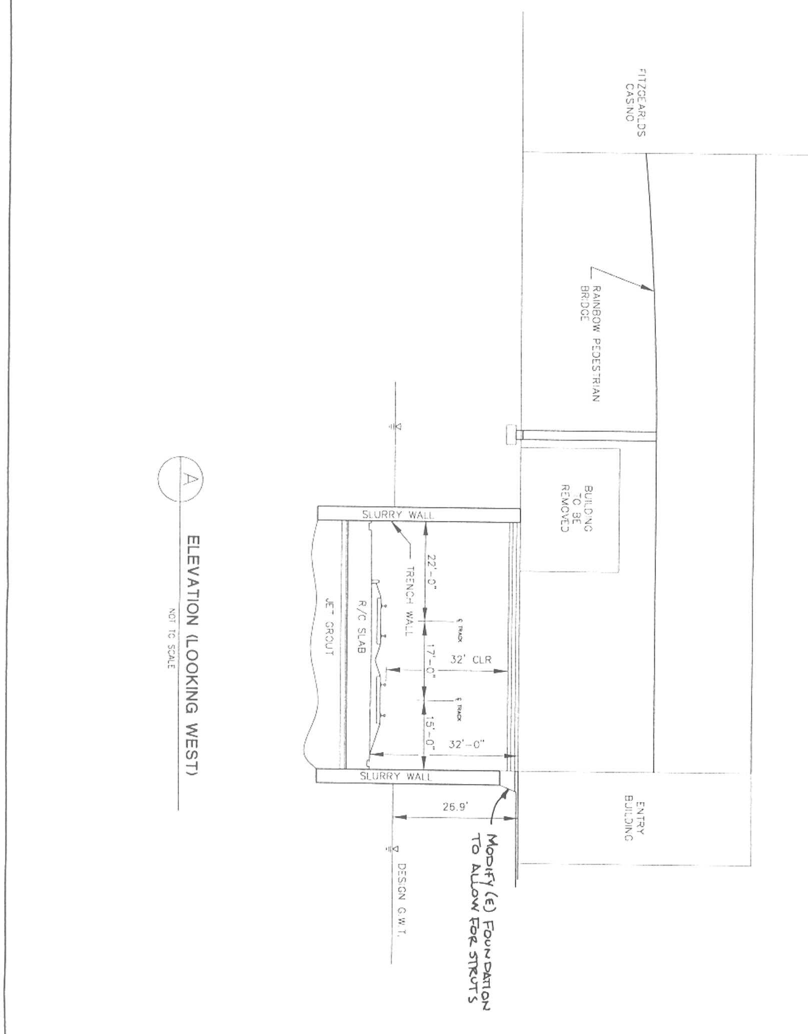
Vice President
***Schnabel Foundation Company***
Farwest Regional Office
3075 Citrus Circle, Suite 150
Walnut Creek CA 94598

Appendix




FITGEARLDS RAINBOW BRIDGE

# RENO RAILROAD CORRIDOR PE

ELEVATION

RENO | WASHOE COUNTY | NEVADA

PREPARED FOR: NDOT

DATE SUBMITTED:

THE NOLTE TEAM
EXPERIENCE TO DELIVER

1-S-002

SA1500

### *10 St. James Avenue*

**Background**
10 St. James Avenue is a new $100 million, 550,000 square foot office complex constructed in Boston's Back Bay. The complex includes a 19-story tower, 50,000 square feet of first-floor retail space and an underground parking facility on 3-1/2 levels for 400 cars. The building was the first Class A office complex to be constructed in Boston in the last 10 years. In November 1998, the owner, Millennium Development Associates, and the construction manager, Lehrer McGovern Bovis Inc., awarded Nicholson Construction Company the general contract for concrete diaphragm wall construction, mass excavation, dewatering, cross-lot bracing, base mat installation and cap beams for the construction of the "box" for the underground parking.

**The Challenge**
Considering the proximity of the new construction to adjacent buildings, minimum movement of the adjacent buildings was a significant challenge. The coordination, time and planning of the excavation and its support by cross-lot bracing were critical. Timing is everything when dealing with soft clays and the control of deformation. The longer sides of the diaphragm wall were immediately adjacent to two buildings, the Liberty Mutual Building (built in the 1950s) and Paine Furniture Building (built in 1913); both were supported on deep foundations that were at a higher level than the final excavation. Of particular note were the heavily loaded belled caissons supporting the Liberty Mutual Building. Another challenging aspect of the project was the tight schedule. Achieving this required planning and coordination of two shifts for construction of the diaphragm wall and installation of the cross-lot braces. Excavation of the basement was performed on an extended single shift of 10 to 12 hours.

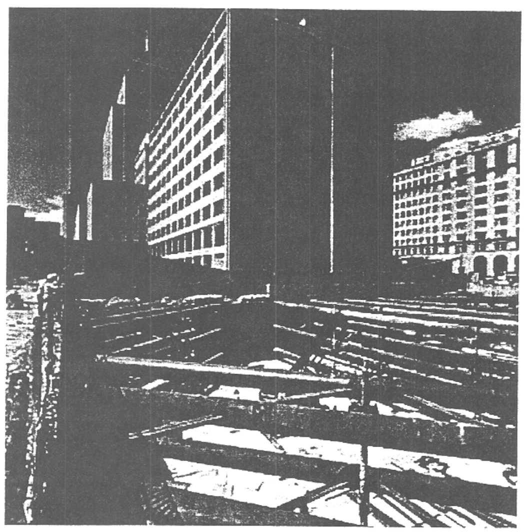
**Meeting the Challenge**
Prior to excavation, bracing, mat construction and eventual erection of the structure, Nicholson installed 42,000 square feet of reinforced concrete diaphragm walls extending to depths of 60 feet.

Work in the wall alignment started with pre-trenching, removal of obstructions (including 165 timber piles), construction of the guide walls, and mobilization of plant and equipment for the diaphragm wall. The 884-foot rectangular perimeter, measuring 271 feet by 171 feet, reaches a maximum depth of 60 feet below grade. The wall is 3 feet thick. Pre-trenching and construction of the slurry wall required the excavation of 8,100 cubic yards of soil and the placing of 4,500 cubic yards of 5,000 psi concrete. Approximately 400 tons of steel were used for the reinforcement of the diaphragm wall. Several problems were experienced during the construction of the wall, mainly due to obstructions encountered below the pre-trenching depth. In spite of the difficulties encountered, the vertical and horizontal movements of the two buildings were well under allowed limits.

The next phase of the project included the excavation of 65,000 cubic yards of soil (3,500 cubic yards of which was contaminated) and the installation of structural steel, including 4,000 feet of 36-inch diameter pipe struts used as cross-lot bracing. The engineer, Haley & Aldrich, identified five different types of contaminated soil, each of which was transported to and treated in a different disposal facility. Up to 2000 cubic yards per day were excavated from the site.

A careful coordination of the mass excavation and installation of the cross-lot bracing was required to restrain the movements of the diaphragm walls below the threshold limit. Upon completion of the mass excavation, at the maximum depth of 45 feet below grade, the 7,500 cubic yard, concrete base mat was constructed. The internal bracing was eventually removed, following the construction of the floors of the underground parking.

Owner: Millennium Development Association
Construction Manager: Lehrer McGovern Bovis, Inc.
Geotechnical Engineer: Haley & Aldrich
Engineer to Nicholson: G.E.I.



*One phase of the project at 10 St. James Street was the excavation of 65,000 cubic yards of soil. A maximum of 100 trucks was used daily to dispose of the soil.*

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### *Exton Square Mall*

**Background**

Exton, Pennsylvania, is a suburb located approximately 20 miles west of Philadelphia. The owner of Exton Square Mall was constructing two parking garages and a new second-story mall level to support the addition of four new anchor stores and over 50 new shops. The initial foundation contractor encountered difficulty in drilling and grouting the highly variable karstic limestone underlying the site. This contractor's method of installation resulted in several pile failures during the load test program. Faced with being six weeks behind schedule, Nicholson was called in on short notice to take over the construction. Two contracts were awarded - one for the support of the new second floor addition and one for the new foundations of the parking garages.

**The Challenge**

Despite the large scope of this project, it was essential that business inside the mall continue without disruption. The construction for the new addition meant drilling inside the mall itself and at close proximity to the perimeter of the building. Work inside the mall had to be done at night after the mall had closed. Merchandise had to be moved and protected. Drilling conditions inside the mall were in areas of tight access and limited headroom (12 feet). Spoils from drilling required special handling. Diverters at the hole were piped through and over the roof of the mall and down into refuse containers on the ground. In addition, construction crews had to ensure the stores were clean and functional by opening time each morning.

**Meeting the Challenge**

This project was ideally suited for the use of Nicholson PIN PILES$^{SM}$ due to the difficult ground conditions and tight access requirements inside the mall. PIN PILES$^{SM}$ were an especially attractive alternative to other types of deep foundations because various drilling techniques can be utilized to advance small diameter casing through virtually any material encountered. Bedrock at the site consisted of karstic limestone with voids and clay seams. The top of the competent bedrock ranged from 20 to 150 feet below the existing ground surface.

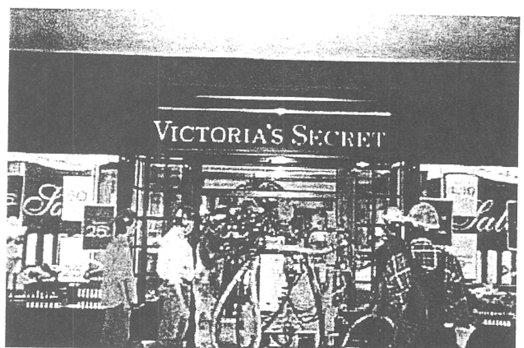
Piles were installed in clusters of three and four to form a pile cap that could carry the load of 450-ton capacity columns. These columns supported the upper addition to the mall. The piles outside the existing mall were installed with large track-mounted drill rigs, while the inside piles were installed with electric powered mini drill rigs. All drills utilized rotary eccentric percussive drilling techniques to advance casing through karstic formations until competent rock was established. For the outside piles, the casing was advanced to the bottom of the bond zone in 10-foot threaded sections. Due to overhead limitations, piles drilled inside were advanced with either 3-foot or 5-foot casing sections.

The maximum design working load was 300 kips in compression. A total of

294 PIN PILES℠ were installed in the interior of the mall and 111 PIN PILES℠ around the perimeter. Average interior pile lengths were approximately 34 feet, ranging up to 150 feet below the existing slab elevation. A total of 355 piles were installed for the new parking garage, with pile lengths ranging from 25 to 85 feet.

**Benefits**
Nicholson's efforts to mobilize within one week and to have the first of five successful piles tested to 300-ton capacity in the second week reflect a superior operational strength. As with most large and technically challenging projects, field conditions did not always meet theoretical expectations, requiring flexibility and the resourcefulness to come up with viable alternatives on short notice. Despite these challenges, the installation of 760 PIN PILES℠ was completed in six months.

Owner and Developer: The Rouse Company
Construction Manager: The Lathrop Company
Geotechnical Engineer: Schiebel Engineering




*The night crew displays the diversity of Nicholson's construction capabilities as they move equipment into one of the stores. Business in the mall carried on "as usual" without any evidence of the nighttime drilling activity.*

Top of Page

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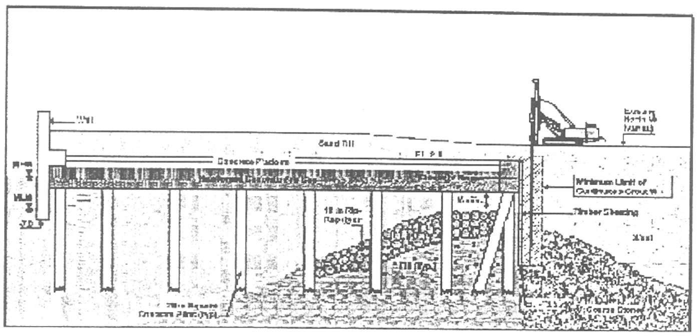
HAYWARD BAKER

KELLER

# Project Summary

## Battery Park City, continued...




*Cross section showing existing deteriorated timber sheeting and location of jet grouted area.*

the timber sheeting. The subsurface profile consists of sand backfill placed over filter stone. This in turn is underlain by a layer of crusher run quarry stone containing cobbles up to nine inches in diameter. This very high porosity material required numerous grout additives and a specific, tightly controlled work procedure to preclude excessive grout loss. For each jet grouted wall, interconnected Soilcrete columns were constructed, by the double-fluid method, to a depth of approximately 20 ft along 800-ft and 500-ft stretches of esplanade, creating effective, 3-ft thick in situ walls.

### Quality Control and Quality Assurance

A very high-strength, corrosion-resistant Soilcrete was needed to meet specification requirements. Extensive pre-construction testing was therefore carried out to assess optimum mix design. Eleven different mixes were tested, using a wide range of cement materials and additives. During construction, numerous in situ samples were retrieved at close intervals at the interstice of Soilcrete columns and tested for unconfined compressive strength, continuity and in situ permeability. This post-construction testing confirmed that the strength requirement in the Soilcrete walls had been achieved.

Both phases of jet grouting were successfully completed without detrimental impact to the park, the existing structures, or the Hudson River.

### Hayward Baker Locations

| | | |
|---|---|---|
| Odenton, Maryland<br>410-551-1980 | Roswell, Georgia<br>770-645-9400 | Ft. Worth, Texas<br>817-625-4241 |
| San Diego, California<br>619-271-1991 | Des Moines, Iowa<br>515-276-5464 | Seattle, Washington<br>206-223-1732 |
| Santa Paula, California<br>805-933-1331 | Palatine, Illinois<br>847-358-1717 | Vancouver, B.C.<br>604-294-4845 |
| Denver, Colorado<br>303-469-1136 | Stoughton, Massachusetts<br>781-297-3777 | Mexico City, Mexico<br>(52-5) 254-2710 |
| Tampa, Florida<br>813-884-3441 | Yonkers, New York<br>914-966-0757 | |

WebSite
www.haywardbaker.com

HAYWARD BAKER

KELLER

# Project Summary

## Jet Grouting

### Battery Park City
### New York, New York

Battery Park City, on the Hudson River, is a combined residential/commercial development built on land 'created' from material excavated during the construction of the World Trade Center. Further development in the 1970's included the construction of a 70-ft wide riverfront esplanade consisting of a reinforced concrete relieving platform supporting several feet of soil. Parallel to the river, the esplanade supports vertical timber sheeting to retain up to six feet of soil. Recent improvements in Hudson River water quality have resulted in an increase in the Teredo Navalis mollusk population. These worm-like, marine borers are now attacking and destroying the timber sheeting.

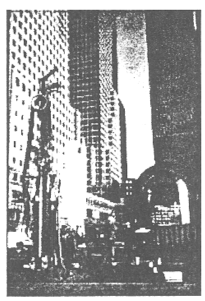
**Selection of Jet Grouting**

Because borer activity would eventually result in loss of soil and surface subsidence, replacing or supplementing the timber sheeting was imperative. However, extensive development of the area, limited workspace and difficult subsurface conditions precluded conventional construction methods. Hayward Baker's jet grouting techniques provided an effective alternative, since jet grouting can be readily accomplished in confined spaces and is effective across the widest range of soil types.

**Phased Construction**

The jet grouting work was completed in two phases. While the first phase work area was relatively open, the second phase was located within extremely restrictive, urban surroundings, requiring special attention to site conditions and spoil containment and disposal.

Project requirements on each phase called for supplementing the timber sheeting with an in situ, jet-grouted structural wall, placed directly behind and in contact with



*Above: Aerial view of the Battery Park City waterfront, with Hayward Baker's Phase I job site, lower center.*

*Left: Hayward Baker's crew, working in tight conditions on the Phase II rehab, predrills in preparation for jet grouting.*

**Owner**
Battery Park City Authority
New York, New York

**Engineer**
Langan Engineering
Elmwood Park, New Jersey

SUBJECT

SA1500

JOB NO. — DPG DESIGNED BY

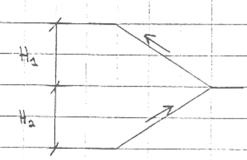
DATE — CHECKED BY

NOLTE

Escalator Weights

$H_1 = 14'\text{-}6''$

$H_2 = 14'\text{-}6''$




* Based on escalator pit dimensions, assume a step width of 24"

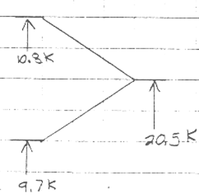
$L = 1.732H + 17'\text{-}10\tfrac{3}{8}''$ (OTIS)

$L = 1.732(14'\text{-}6'') + 17'\text{-}10\tfrac{3}{8}''$

$\Rightarrow L \simeq 43'$

$R_B = 213(43) + 517 = 9{,}676^{\#}$ (Vertical reaction @ bottom support)

$R_T = 213(43) + 1686 = 10{,}845^{\#}$ (Vertical reaction @ top support)



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## Column Loads

Column 1:

230 sf (120 psf DL) = 27.6 Kips
230 sf (100 psf LL) = 23.0 kips

Column 2:

237 sf (120 psf DL) + 800# HVAC = 29.2 kips
237 sf (100 psf LL) = 23.7 kips

Column 3:

236 sf (120 psf DL) + 800# HVAC = 29.1 Kips
236 sf (100 psf LL) = 23.6 Kips

Column 4:

264 sf (120 psf DL) + $\frac{67.7K}{2}$ + 1300# HVAC = 66.8 kips
264 sf (100 psf LL) + $\frac{69.9k}{2}$ = 61.4 kips

Column 5:

198 sf (120 psf DL) + $\frac{67.7K}{2}$ = 57.6 Kips
128 sf (100 psf LL) + $\frac{69.9K}{2}$ = 47.8 Kips

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Column 5 (+ ½ bridge load)

roof

(9'-1")(7'-0") = 64sf

2nd floor (9'-1")(7'-0") = 64sf

Mezzanine (9'-1")(7'-0") = 64sf (light load, no LL)

Total = 128sf (LL+DL) 192sf (DL)

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Column 3 +800#

roof $(5'\text{-}0'' + \frac{23.5'}{2})(9'\text{-}1'') = 152\ sf$

2nd floor

$(5' + \frac{23.5'}{2})(5'\text{-}0'') = 84\ sf$

Mezzanine ~0

Total = 236 sf

Column 4 (+1300# - ½ bridge load)

roof $(\frac{23.5'}{2} + \frac{14'}{2})(9'\text{-}1'') = 170\ sf$

2nd floor $(5'\text{-}0'')(\frac{23.5}{2} + \frac{14'}{2}) = 94\ sf$

Mezzanine

~0

Total = 264 sf

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JOB NO. DESIGNED BY

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Column 1

Roof $\left(\frac{5'\text{-}1'' + 4'\text{-}6''}{2}\right)(3'\text{-}0'') + 9'\text{-}0''(5'\text{-}1'' + 4'\text{-}6'') = 101\ sf$

2nd floor $\left(\frac{9'\text{-}7''}{2}\right)(9'\text{-}0'') = 43\ sf$

Mezzanine $\left(\frac{9'\text{-}7''}{2}\right)(9'\text{-}0'') = 86\ sf$

Total = 230 sf

Column 2 7800#

roof $(9'\text{-}0'' + 5'\text{-}0'')(9'\text{-}1'') = 127\ sf$

2nd floor $(9'\text{-}0'' + 5'\text{-}0'')\left(\frac{9'\text{-}7''}{2}\right) = 67\ sf$

Mezzanine $\left(\frac{9'\text{-}7''}{2}\right)(9'\text{-}0'') = 43\ sf$

Total = 237 sf

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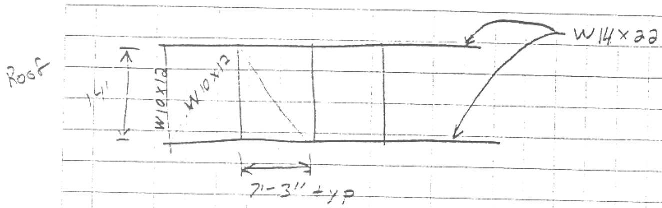
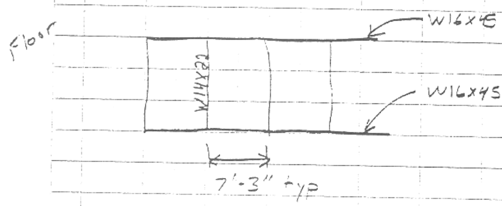



5 bays

height ~ 16.5'

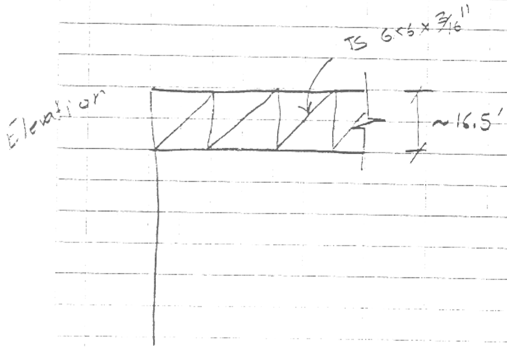
EIFS Panels

single ply roof

3½" lght wgt conc

over 2"x20 ga deck

∴ say 5½-6" lgt. conc.



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Dead Loads:

Floor steel ~ 100#/' = 6.25 psf

Roof steel ~ 140#/' = 8.75 psf

Wall steel ~ 150#/' = 9.4 psf

Single plywood roofing ~ 10 psf

Floor core load ~ 6(10 psf) = 60 psf

Walls ~ 20psf/wall) = 330plf = 20.6 psf

DL = 115 psf

Total dead load = 115psf (16')(73'-7") = 135,393#

Dead load reaction at building = 67.7 k

Total reaction at building = 67.7k + 69.9k = 137.6 k

Total reaction = 137.6 k

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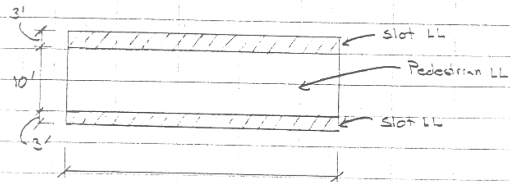
Pedestrian Bridge Loads

Live Loads:

Pedestrian LL = 100 psf

Slot machine LL = 150 psf (3' strip each side of bridge)




Total LL = (3' + 3')(73'-7")(150 psf) + 10'(73'-7")(100 psf)
= 139,808#

Load to building = 69.9$^{k}$ (½ total load)